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**UNITED STATES ENVIRONMENTAL PROTECTION AGENCY**
REGION 5
77 WEST JACKSON BOULEVARD
CHICAGO, IL 60604-3590

US EPA RECORDS CENTER REGION 5
471762

REPLY TO THE ATTENTION OF:

**March 4, 1997**

VIA FIRST CLASS MAIL
John Seymour
Woodward-Clyde Consultants
38777 W. Six-Mile Road, Suite 200
Livonia, MI 48154

**Re: Albion Sheridan Landfill Superfund Site**
**Notification of Disapproval of Preliminary Design**

Dear Mr. Seymour:

The U.S. EPA and Michigan DEQ have reviewed the preliminary design for the Albion-Sheridan Landfill and we are unable to approve the document at this time because it does not comport with the Record of Decision. Listed below are our comments which we would be willing to discuss with you via a conference call if that is your desire.

General Comment

USEPA does not concur that the landfill cover system proposed in the 30% Design is a "technical equivalent" to that in the described in the ROD and SOW. The purpose of a drainage layer is to shed water immediately off of the FML or barrier layer. In order to accomplish this, a *continuous* drainage layer is required over the entire surface of the FML or barrier layer.

As proposed, the strip drains are located 20 feet apart. In between the strip drains, there is the potential for as much as 1.7 feet of head to build up on the FML (as concluded by the Help Model provided in the February 14, 1997 letter to U.S. EPA). Pinhole leaks on the FML will be enlarged in areas where there is 1.7 feet of water ponding on the FML which will in turn, cause generation of leachate and compromise the integrity of the FML. Roots from the overlying vegetation will seek out the 1.7 feet of water ponding on the FML which will in turn encourage vertical cracks within the cover soil layer.

Specific Comments

Page 1-4, paragraph eight: Please clarify that Woodward Clyde is working on behalf of the liable party group.

Page 1-7, Section 1.3.3 Site Surveying: Please use the Global Positioning System (GPS) for well

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locations.

Page 1-8, Section 1.3.6 Air Emissions Study: Please include field verification of the landfill air emissions estimation model to confirm that all ARARs are met during landfill remediation and waste consolidation activities.

Page 2-1, Restrictive Covenants/Deed Restrictions: As discussed in our February 4, 1997 meeting, it will be necessary to execute restrictive covenants that are administratively similar to the MDEQ's model documents. The 30% design document states that deed restrictions or a local ordinance will be implemented 30 days after the approval of the pre-design studies report, this was not done.

Page 2-2:The PreDesign indicated that the use of native plant species has "substantial merit" and that the "study results will be incorporated in the Remedial Design phase." The discussion in the 30% appears to leave some doubt whether or not native species will be used.

Page 2-2, Drummed Waste: The contractor hired to complete the drum removal activities should determine if excavated drums are "structurally sound."

Page 2-4: Change bullet at the top of the page to read that the air monitoring will be required "...during <u>and after</u> the remedial action...." This is consistent with page 5 of the SOW.

Page 2-4: The section titled "Contingent Remedy" should include actions to be taken in the event landfill gas concentrations exceed applicable State criteria. The contingency should be consistent with Act 641 R.299.4433(4) and 40 CFR 258.23 which are specifically for methane; however, can be easily reworded to include other compounds or groups of compounds relevant to this site. Page 6 of the SOW specifically states that "corrective measures" will be developed in the RD for addressing exceedances of the action levels at the fence line.

Page 3-3: The material specifications are not "arbitrary." The material specifications are derived from Michigan's Act 641. Also, in Section 3.3.1, change the reference of "with the required two percent slopes" to "to a minimum of a four (4) percent slope."

Page 3-3: Second paragraph, Section 3.3.1 - Change the reference of "with the required two percent slopes" to "to a minimum of a four (4) percent slope."

Page 3-3: Third paragraph, Section 3.3.1 - The site material described in the RD appears to be sufficient for the daily cover; however, it does not meet the requirements of the gas collection layer. Geotechnical testing should be performed on the all proposed material to substantiate whether or not the materials are equivalent to what is required by the ROD.

Page 3-4: Section 3.3.2, first paragraph - Remove quotation marks from improper ROD quotation. Also, the soil permeability and transmissivity standards stated in the ROD and SOW are appropriate for the drainage layer of the cover system even though the particular standards are cited in the leachate collection section of Act 641. The reason it is applicable is that both layers serve identical functions as far as transporting water off the FML.

Table 3-1: Please include Michigan Act 641 as an ARAR under the Groundwater Monitoring and Analyses section. This ARAR requires additional monitoring than what is proposed in the 30% design.

Section 3.3.2; Paragraph 1: The 3/8-inch maximum grain size specified in the ROD is appropriate for direct contact with the FML to prevent punctures. Larger materials can be used if a geotextile (8-12 oz./sq. yd.) is installed directly above the FML. For <u>any</u> drainage layer to be effective over the long term, a geotextile should be installed <u>above</u> the drainage layer to prevent soils from clogging it. Installed cost for an 8 oz. geotextile runs about $5700 per acre, non-union (quote from National Seal).

At the end of this paragraph, and in paragraph 2, the consultant (WCC) seems to be making some sort of appropriateness determination for the transmissivity performance standard in the ROD. Regardless of the regulatory derivation, transmissivity (T) <u>is</u> an appropriate alternative design criterion to hydraulic conductivity ($k$) for a synthetic drainage layer. Since the ROD specified a minimum T it should be considered a performance standard that must be met or exceeded. If the PRPs wanted to comment about this, the public comment period was the appropriate time.

Page 3-4: Section 3.3.2, third paragraph - Again referring to the leachate collection section of Act 641, R.299.4423, a $1.0 \times 10^{-2}$ cm/sec fill may be replaced with a $1.0 \times 10^{-3}$ cm/sec fill when used in conjunction with a synthetic drainage material. Alternatively, a synthetic drainage system, such as a geonet, which is capable of providing a minimum transmissivity of $3 \times 10^{-5}$ $m^2$/sec over the <u>entire area</u> of the FML will meet the requirements of the ROD. The drainage system WCC has proposed does not provide adequate drainage over the entire cover system.

Page 3-4: Section 3.3.2, third paragraph, last sentence - If spreading a 6-inch drain layer cannot be accomplished without reasonable protection of the FML, then it is incumbent upon WCC to modify the design to specify an appropriate depth of the drainage layer.

It's interesting that the 30% design proposes strip drains 1.5 inches thick, and the 2/14 letter downsizes them to 1.0 inch. This must be due to the process of materials optimizing (i.e., cost cutting). The design states that they have spaced the strip drains to allow for maintaining the intermittent saturated depth of less than 16 inches (the term refers to the peak daily head, or saturated thickness following a 25-year, 24-hour storm event). An efficient drainage layer will maintain a peak daily head within its own thickness. (Note comments below on their HELP model output. In the 2/14 letter, the peak daily head has crept up to 20 inches because WCC apparently caught the model errors in the PRPs 30% submittal.)

I agree that a 6 inch granular drainage layer is insufficient from a constructability perspective. Even using low ground-pressure earth moving equipment, 12 inches should be the minimum. There is no reason other than cost that the design could not increase the thickness of the granular materials instead of substituting less permeable soils, as long as the *k* standard is met or exceeded. However, from the alternative design proposed, it is clear the PRPs do not want to take this direction.

3.3.3 Passive Gas Venting System: In general, I don't understand why the PRPs think we should consider lowering the standard for the gas collection layer from sand to a less permeable soil.

Page 3-5, Section 3.4 Drum Removal: Please expand on the drum staging area. The staging area should be a storage pad lined with PVC or PE and bermed to contain potential spills/leaks.

Page 3-5: Section 3.3.4, second bullet - 5.2 x $10^{-4}$ cm/sec material in conjunction with strip drains is not technically equivalent to 1.0 x $10^{-3}$ cm/sec material.

Page 3-5: Section 3.3.4, third bullet - 5.2 x $10^{-4}$ cm/sec material does not qualify as a "permeable soil layer" as required in Rule 425 of Michigan's Act 641. Good engineering practice dictates the use of a material with a minimum permeability of 1.0 x $10^{-3}$ cm/sec.

Page 3-5: Section 3.4 - Make this section consistent with page 23 of the ROD and page 2-2 of the RD. Specifically, "drums containing <u>liquids and/or</u> hazardous materials" require off-site disposal or treatment. The RD does not specify how the USTs located on the east side of the site will be dealt with. Please do so.

Page 3-5: Section 3.5 - Change the third bullet in this section to read "Remove all waste and visibly stained soils."

Page 3-5: Section 3.5 - The statement in the sixth bullet of this section makes a statement about an action level at the perimeter of the exclusion zone. The HASP must distinguish between perimeter action levels and breathing zone action levels. Also, these action levels must be established based on the acceptable action levels of the constituents identified at the site.

Page 3-6: Section 3.6 - Add a fourth bullet stating the following: "Comply with applicable air quality standards."

Page 3-6: Section 3.7 - USEPA does not agree that the cover system proposed in the 30% Design is equivalent to the cover system required in the ROD. Please see General Comment.

Page 3-6: Section 3.7, third bullet - Will compaction requirements be increased for areas directly beneath of the service road?

Page 3-6: Section 3.7, Drainage Layer - Act 641 does not require 24 inches of drainage layer. Also, the statement in the second bullet is not believed to be true.

Page 4-1: Section 4.2, paragraph 2 - Change the sentence that starts out "Any large trees..." to "Trees and shrubs..." The term large is too subjective. Also, all wooded vegetation should be chipped to ensure that unacceptable voids are not created. The RD does not specify how the USTs located on the east side of the site will be dealt with. Please do so.

Page 4-1, Section 4.2 Waste Movement and Site Grading: Please include language to address metal debris to be scrapped off prior to initiating grading activities.

Page 4-3: Section 4.4.1 - Change title to "Gas Venting/Foundation Layer." Silty sand is not acceptable for this layer. Also, will compaction requirements be increased for areas directly beneath of the service road?

Page 4-3: Section 4.4.2 - It is the PRP's responsibility to ensure that the materials used for construction of the cap are suitable for ensuring slope stability during and after construction including saturated conditions prior to establishment the vegetative cover.

Page 4-3: Section 4.4.3 - The cover soil is to be placed above the drainage layer, not the FML.

Page 4-4: Section 4.4 - The topsoil/vegetative layer should be dissimilar enough from the cover soil so as to discourage root growth into the underlying cover soil layer.

Page 4-5: Section 4.5.2 - Does this comply with the manufacturer's minimum recommendations?

Page 4-6: Section 4.5.4 - Analysis of the storm water runoff analysis must be part of the RD before the Final RD can be approved.

Section 4.5.5, and Appendix A, HELP Model - One of the critical outputs from the HELP model is the average annual head over the barrier layer. There is a strong correlation between average annual head and the amount of water that percolates through the barrier layer. The 2/14 HELP model shows an average head of about 12 inches. This is pretty high. A good drainage system should not allow the maximum head (peak daily head) to exceed 2 inches, and the average head to exceed 1 inch. This can be easily accomplished with a full-area geonet, or a gravel layer 12 inches thick or greater. The synthetic cap profile at one of my sites results in about 0.35 inches of average head.

In paragraph 3, the PRPs point out that the HELP model results show no percolation/leakage through the FML, and minimal head build-up even after a peak daily storm event. They use the output from the model to calculate inflow rates for purposes of spacing their strip drains. Just a comment on the two HELP model runs provided in Appendix A. Both model runs show less than one inch of water buildup over the FML both for average annual and peak daily heads, and zero infiltration through the FML. When zero infiltration occurs, either your drainage layer is of excellent design (not the case here) or something is wrong with the model input. In this case, values for slope and drainage length were input as zero. The model reads this as if all the water is almost immediately drained out once it hits the drainage layer. Slope length of zero will nearly always result in zero infiltration across the barrier layer. This input error, which lead to erroneous conclusions in sections 3 and 4, was <u>not</u> carried forward to the model results provided in the 2/14 letter. The peak daily head with the drains increase from 16 to 20 inches, yet, the consultant draws the same conclusions in that letter as are reached here.

The PRPs should provide HELP model runs that include 1) a granular drainage layer based on the minimum *k* performance standard and 2) a drainage layer composed of the maximum allowable grain size of 3/8 inches (this grain size corresponds to pea gravel. With pea gravel you can expect a *k* of about $3 \times 10^{-1}$ cm/sec or better, which would substantially reduce the average head and bring annual average percolation through the barrier layer to less than 1/10 inch.). The PRPs have made comparability claims for their alternative without providing these model runs for comparison.

Page 5-2: Add Specification Section 01300 - Submittals.

Page 6-1, Section 6.1 Introduction: The deed restrictions should prohibit the installation of groundwater wells and *use* of groundwater from the area noted in Figure 4 of the ROD.

Page 6-2: Section 6.4 - Is an air discharge permit required or a substantive requirements document form the State? How will PRPs ensure compliance with substantive discharge requirements?

Page 6-2, Section 6.4 Permit Requirements: The liable parties will need to petition the MDEQ's Surface Water Quality Division for a mixing zone determination to determine if contaminated groundwater is venting to the Kalamazoo River.

Implementation Schedule - Need to assume that the Gas Vent/Foundation layer will need to be protected to avoid potential erosion from rain and snow melt.

Page 11-1:Section 11.1 - Act 641 requires a minimum of quarterly monitoring for methane. All references to "periodic" inspections or monitoring should be changed to "routine." The O&M should include provisions to address settlement of the cover as well as erosion.

Page 11-1, Section 11.2 Groundwater Monitoring System Operation and Maintenance: Additional investigative work is needed to complete the basic hydrogeologic study for a landfill site as required in the Solid Waste Management rules of Act 451. Additional information needed to support the groundwater monitoring proposed in the 30% design report needs to include: 1) background water quality on the east and possibly west sides of the landfill; 2) the defined aquifer thickness; 3) the defined deep bedrock groundwater quality and bedrock elevations across the site and; 4) a map that shows the distance to all existing wells and the properties in the surrounding area that have potential for groundwater supplies. The map needs to identify all soil borings and wells with ½ mile of the site, including all domestic, municipal, industrial, oil and gas wells for with copies of logs are available. This area includes wells south of the Kalamazoo River; 5) include or reference the location of observation well records or soil borings; 6) provide a groundwater elevation map using elevations referenced to United States geological survey datum.

This data should include possible variations in groundwater flow direction; 7) provide or reference geologic cross-section or fence diagrams; 8) provide a list of all stabilized static water level readings; 9) a monitoring plan for the surface water to that may receive leachate by groundwater venting; 10) all wells must be clearly labeled and shall be properly vented, capped, and locked when not in use. In addition, all wells shall be visible throughout the year. Protective posts painted in bright colors will help comply with this request; and 11) the construction of monitoring wells will be completed by a well driller registered under or regulated by Act 451.

Page 11-2:Section 11.3 - As per previous comment, gas monitoring is required. The modeling performed as part of the PreDesign was to determine whether or not an active or passive gas system was necessary. It was not a substitute for air monitoring during remedial action or O&M. Furthermore, the modeling was performed on a vertical well gas network not the proposed horizontal well network. As such, the 30% Design is deficient because it does not include an air monitoring program as required by the ROD.

Drawing 3: Identify location of Sections A, B, C, D, and E shown on Drawing 4. Grading in some areas, especially to the north, of the FML appear to be greater than 25 percent.

Drawing 4:We do not agree that Sections C, D, or E portray a cover system that is in compliance with the ROD.

Drawing 5:Is the note on the drawing that states "Manhole See Detail" referring to the "Catch Basin" shown on Drawing 6.

In the February 4, 1997 conference call, we raised the issue of the location of the infiltration basin. WWC provided a response in their February 14, 1997 letter to U.S. EPA. U.S. EPA has reviewed the response and does not believe that it addressed our original concern. How will this infiltration basin impact water levels within the fill? We believe that water infiltrating through the basin will intersect the groundwater table and will flow with groundwater through the fill itself. During a storm event used in the "Response to Technical Issue 5", the amount of water could cause a rise in the water table elevation. With time, the infiltration pond will likely start to silt up. This would cause standing water to collect in the pond. Standing water would cause mounding to occur which would result in the water table to rise up into the fill more than shown on the cross-section.

Drawing 6:We do not agree that Sections A, B, and C portray a cover system that is in compliance with the ROD. Section A refers to "foundation layer", not a gas collection/foundation layer. Was this intentional?

Drawing 7:This drawing does not portray a 200 foot spacing for the horizontal vent well system as described in the text. In addition, U.S. EPA recommends a N-S interconnected horizontal vent pipe at the crown of the cover system. This may affect the placement of the road. Detail 1 on Drawing 7 does not match Detail 1 on Drawing 8.

Drawing 8:Detail 2 should show slots into the Gas Collection layer to collect the accumulated gas.

Drawing 9:The chain link fence must include three strands of barbed wire per the SOW.

APPENDIX B FINAL REPORT PERFORMANCE MONITORING PLAN

Page 1-1, Section 1.1 Site Location and Description: The Amberton Village housing development is located on the east side of the site with residences about 500 feet away from the landfill. Drawing 2 identifies a number of lotted properties located immediately adjacent to the landfill. It does not appear that these lots have existing residences. Although there will be restrictive covenants preventing water supply wells downgradient from the landfill, will there be any isolation distances required? Lateral isolation distances for landfill are required under the Michigan Solid Waste Rules.

Page 2-1, Section 2.1 Site Geology: Please reference and/or include the location of the geologic cross-section or fence diagrams.

Page 2-1, Section 2.2 Hydrogeology: There needs to be adequate definition of groundwater flow direction before determining a groundwater monitoring plan. The monitoring system will be based on locations designed to assess the impact of the discharge on groundwater. The state solid waste programs generally require groundwater monitoring wells at least every 150 to 300 feet for monitoring purposes at landfill that have a base liner, sidewalls, and perimeter collection systems. The Albion-Sheridan Township Landfill, without a liner or sidewalls, and only a cap proposed for its remediation, has spaces of over 1000 feet between wells for groundwater flow direction determination and monitoring. Listed below are our suggestions on how to correct this issue.

On the ***northwest side***, there is approximately 1,050 feet between MW01 and MW03. The groundwater flow direction maps show different curvatures on the contour lines from 1992, 1993, and 1996. These discrepancies confuse whether the contour lines flatten out, extend more to the north or curve back to the south. By installing wells in this vicinity, it will help determine if any contamination may be migrating from any areas upgradient of the site. If the adjacent landowner will not permit well installation, one important location that should be addressed would be just within lot 27 parcel 4 about 400 feet north of MW03. The nested wells should be screened in the unconsolidated glacial, weathered bedrock and shallow bedrock aquifers. This location will be good for obtaining additional groundwater quality information to support the contaminant plume profiles provided in June 1993. It would also be helpful to know for monitoring purposes if all contaminant plumes end just north of MW03 as shown on the diagrams or just short of MW01, 1000 feet north of MW03. If the groundwater flow direction is more westerly in this area then it needs to be adequately monitored.

On the ***west side,*** it would be helpful to include a well cluster between MW03 and MW04 for downgradient monitoring. There is more than 450 feet between MW03 and MW04.

On the ***south side,*** arsenic has been detected in MW16SB at 7.9 ug/l. There are no wells at the appropriate depth downgradient from MW16SB to monitor the arsenic plume. Adding wells screened in the weathered bedrock and shallow bedrock would be useful to monitor the downgradient plume at MW12SG and MW13SG. The MDEQ would also like to see additional wells screened in the weathered and shallow bedrock at MW10SG.

On the ***southeast side,*** please include two additional well clusters between MW05 and MW07. These wells could be an important downgradient location for all aquifers that have been overlooked.

On the ***east side***, given the lack of data along nearly the entire side, groundwater quality for homes immediately adjacent to the landfill can not be assured. There is no protection for residences such as a 300 feet isolation distance. Currently MW02 and MW05 are spaced approximately 300 feet apart. Any wells placed on the perimeter between MW02 and MW05 would be an improvement.

The ***river***. The liable parties charge that groundwater is discharging to the Kalamazoo River based on an upward gradient at a well nearly 700 feet north of the river. Groundwater contaminant data does not appear to support this theory. Additional information such as groundwater flow direction on the other side of the river, vertical gradient determination on the opposite side of the river, and tracer test information to support the liable parties claims is needed. The contaminant concentration profiles dated June 1993 do not support plume discharge to the river. Groundwater samples results from MW16SB show 7.9 ug/l arsenic at a depth similar to the previously mapped plume depth. The plume is not getting shallower to allow venting to the river. There is no monitoring planned south of the river and there are no wells south of MW16 that are screened a plume depth (weathered or shallow bedrock aquifers). If groundwater is venting to the river, it will be necessary to include a discharge permit within the permit requirements section. At a minimum, it will be necessary to collect upgradient and downgradient surface water samples.

Page 4-1: Section 4.1 Excavation

This section does not address removal or closure of the USTs. Please do so.

Page 4-1: Section 4.2 Drum Removal and Characterization, Paragraph 2

References using an AZCAT test. The text should spell out what the acronym means.

It may be helpful to illustrate a flow chart or other diagram showing the initial characterization regime , how the waste will be segregated and how the waste will end up in each of the final disposal options.

Page 4-1: Section 4.3 Temporary Storage and Transportation

Section 4.3 should be more specific in where the drums will be stored on the surface of the landfill.

Page 4-2, Section 4.4 Monitoring Requirements

As stated previously, air monitoring during remedial action is required by the ROD.

Page 4-2, Proposed Section 4.7

A reporting section should be added here to report or summarize the drum removal activities.

Page 5-1, Section 5 Landfill Cap Construction Monitoring Plan: The performance monitoring plan includes monitoring the construction of the landfill cap but not long-term performance monitoring of the landfill cap. Please expand this section to include long-term performance monitoring of the landfill cap.

Page 6-1, Section 6.0 Landfill Gas Collection System Monitoring Plan

As stated previously, a remedial action and O&M landfill gas monitoring program is required.

Page 6-1: It will be necessary to field verify the landfill air emissions estimation model calculations and prepare a long-term operations and maintenance plan if necessary.

Page 7-1, Section 7.1 O&M Monitoring Well Locations

Please provide a figure showing exactly which wells will be sampled in the quarterly and annual groundwater monitoring program.

Page 7-1, Section 7.1 O&M Monitoring Well Locations: The locations of monitoring wells should be revised or expanded once further hydrogeologic definition is provided, see comment no.15. Existing monitoring wells for shallow glacial groundwater should include MW06SG (to monitor the existing arsenic plume), MW12SG and MW13SG (to monitor for arsenic in downgradient wells). Weathered bedrock wells need to be included in the monitoring plan. This highest concentration of arsenic is in MW06WB. This well is exactly downgradient and screened in the best location to show downgradient groundwater quality. The monitoring wells selected for weathered bedrock should be identical to the shallow bedrock wells.

Page 7-1, Section 7.2 Monitoring Well Installation: Please clarify if schedule 40 or 80 PVC riser and well screen will be used.

Page 7-2, Section 7.4 O&M Groundwater Analysis Program: Based on existing information, the monitoring plan should be modified to include additional parameters appropriate for a landfill that has accepted industrial and municipal waste.

Quarterly groundwater monitoring needs to include chlorides, iron, sulfates, total inorganic nitrogen, total dissolved solids, magnesium, manganese, potassium, sodium, as well as, field parameters, arsenic and ammonia. In addition, the groundwater depth and elevation before purging will need to be collected for all site wells.

The quarterly monitoring of the seven drinking water wells will need to consist of: 1) all parameters listed above; 2) parameters listed in Part 115, P.A. 451 as follows: heavy metals as listed in R229.4452 including aluminum; primary volatile organic constituents (VOCs) listed in R299.4452 (halogenated and aromatic VOC's); secondary organic parameters as listed in R299.4454 (carbon disulfide and 1,2-Dibromo-3-chloropropane); cyanide, mercury, antimony, and the parameters included in the groundwater monitoring plan.

Annual monitoring will need to consist of the parameters listed for the quarterly monitoring of the seven drinking water wells.

Page 7-2, Section 7.4.2 Annual Groundwater Monitoring: Please include iron and bis(2-Ethylhexyl)phthalate as chemicals of concern to be monitored for.

Page 7-2, Section 7.4.3 Five-Year Review Groundwater Monitoring: Post-closure care included in monitoring of a Type II (lined) landfill in Michigan under the part 115 rules of Act 451 (previously Act 641) continues for not less that 30 years. Groundwater monitoring proposed for the Albion-Sheridan Township Landfill will cease after five years if arsenic is at an acceptable level. Consideration should be given to extending this requirement.

Section 7.3 O&M Groundwater Sampling Program

References to Act 641 and Act 64 should be revised to the appropriate parts under NREPA , Act 451.

Please reference figure requested in previous comment.

Please provide a schedule for quarterly and annual groundwater monitoring.

Please provide a table which summarizes which wells are analyzed for what parameters on what schedule.

Please provide a table summarizing the parameters to be analyzed, methods and detection limits.

Page 7-2, Section 7.4.3 Five-Year Review Groundwater Monitoring

What the will the comparison criteria be? Michigan Public Act 307 which was in effect at the time the ROD was signed? Natural Resources and Environmental Protection Act 451, Part 201 which is in effect now? Applicable criteria at the Five-Year mark? Residential versus industrial?

Page 8-1, Section 8.2 Analytical Methods: Please include the use of Operational Memorandum #6, Revision #4 dated September 13, 1995.

Page 8-1, Section 8.3 QA/QC

Will the O&M QAPP be provided in the next submittal?

Page 8-1, Section 8.5 Requirements for Health and Safety Protocols

Should references to Act 641 and Act 64 be revised to the appropriate parts under NREPA , Act 451?

Page 9-1 Reporting

This section needs to be more explicit.

At a minimum, analytical results should be summarized in tabular format cumulatively. IE. the tables should include all quarterly data (current and historic) for purposes of comparison. Exceedances of applicable criteria should be highlighted.

For the compounds of concern, it would be useful to plot concentrations vs. time for specific wells. This would be useful in determining if the concentrations are increasing with time and if this is a concern.

Isoconcentration maps for arsenic would be beneficial.

Groundwater contour maps for each aquifer/each quarter would be most beneficial.

A schedule for reporting the results to the U.S. EPA and MDEQ should be provided or described.

Please detail what information will be provided in the report.

Page 10-1, Schedule

Page 4 of the Statement of Work requires quarterly monitoring for five years. The language in this section implies that there is "wiggle room" to modify the sampling schedule after the first year. This is not the case. Furthermore, the parameters to be analyzed quarterly and annually is spelled out in detail in the SOW.

<u>SOPs</u>

SOP-08

Section 5.0 Handling of Decontamination Fluids

USEPA does not recommend placing equipment decontamination fluids upgradient of the landfill. This water will have the potential to be transported with groundwater through the refuse.

**Letter of February 14**

This letter provides an analysis of the proposed strip drain alternative to the drainage layer, using corrected HELP model output.

**Page 2**

**Paragraph 1** - The ROD set a minimum standard for $k$ of $1 \times 10^{-2}$ cm/sec. It is not up to us to lower that standard short of issuing an ESD, whether or not the State would accept it. In truth, a sand layer of $1 \times 10^{-3}$ cm/sec will not perform much better than the soil they are proposing. It just isn't permeable enough, and 6 inches is not thick enough.

**Paragraph 3** - This paragraph makes no sense, and the claim of a 2 inch "maximum average saturated soil depth" (I assume that means head) over the FML cannot be substantiated from the attached model run. When dealing with soils with a $k$ of $5 \times 10^{-4}$ cm/sec, the key output affecting infiltration through the FML is the average annual head over the FML, which according to the attached HELP model is high at 12.5 inches. Also, in the text of the letter and the design document the on-site soils are represented as having a $k$ of $1 \times 10^{-4}$ cm/sec, whereas the HELP model assumes a $k$ of $5 \times 10^{-4}$ cm/sec. This difference will also contribute to a lower output for percolation through the barrier layer and make the proposed design alternative look better than it might otherwise be. Conversely, if the on-site soils actually do have a $k$ closer to $1 \times 10^{-4}$ cm/sec, the average annual head will increase, leading to increased percolation.

A related comment is that QC for the on-site soils during construction will probably be minimal. Unless the degree of homogeneity of these soils has previously been determined, the resulting cap may contain areas where the $k$ is lower than the design $k$. This would result in sub-areas of the cap that support a higher average annual head than that already indicated in the model., (with or without the strip drains).

**Page 3**

**Paragraph 1** - WCC has judged that the strip drain approach is applicable because it is similar to how leachate drains are developed for landfill bottom liners. The model used by WCC assumes parallel channels for leachate collection use. In a real situation, however, the drain strips in a leachate collection system will converge at the center from a high point at the outside edge of the landfill. Therefore, when water (or leachate) flows downhill toward the center of the "bowl", it will naturally encounter a drain and be moved efficiently to a collection area. If the strip drains are installed for cover drainage as depicted in Figure 6 of the 30% design, there is no convergence of the drain strips. A substantial amount of water in the soil will not be collected by the drains, resulting in higher average annual head over the FML than would be that case if an efficient drainage layer were installed over the entire area.

In the last sentence WCC compares the peak daily saturated thickness (with drains) to that of a six inch drainage layer with a *k* of $1 \times 10^{-3}$ cm/sec. This comparison is not valid because that *k* value does not meet the minimum performance standard for *k* in the ROD (see comment in re: page 2, paragraph 1 above). WCC needs to compare their proposal with a *compliant* alternative.

WCC has not factored in a reduction in strip drain efficiency due to clogging over time of the geotextile that wraps the strip drains. Such clogging does happen, and will happen more readily where large volumes of water carrying soil and silt particles pass through a relatively small surface area. For the strip drains there is approximately a one-foot square surface area through which water from 20 square feet of drainage area needs to pass, as compared to a 1:1 ratio of drainage area to geotextile area for a full-coverage drainage layer.

**HELP Model Attachment** - The input assumptions for the FML are fairly optimistic. The U.S. Army Corps of Engineers (USACE), which developed the model, usually assume 5 to 7 defects per acre. In the HELP manual the range for installation defects per acre is from 0 to 1 for excellent placement, 1 to 4 for good placement, 4 to 10 for fair placement. WCC has optimistically assumed the high end of good placement, which results in a fairly low infiltration rate even with the 12.5 inch annual head over the FML. Infiltration will increase with increasing defects.

In summary, the design analysis used to support the proposed alternative cap profile appears to have been based on optimistic input assumptions to all areas that would support the analysis, and on less than allowed minimums for areas from which the PRPs want to change. The result still does not look as good as an alternative that uses a full-area drainage layer. An efficient, full-area layer will eliminate any concerns about slope stability, even with the relatively high slopes at this site, and will virtually eliminate percolation through the barrier layer. Costs for a 12-inch gravel drainage layer will vary locally. A geonet will cost approximately $7900 per acre, installed, non-union labor. The full synthetic profile (FML, Geonet, Geotextile) will cost about $26,500 per acre. If the remaining soils are available on-site as represented, assume $1.50/cubic yard for excavation and placement. Twenty-four inches of select fill and 6 inches of topsoil would cost about $4840 per acre.

Sincerely,

Jon W. Peterson

Jon Peterson, RPM
Section #6

**U.S. ARMY CORPS OF ENGINEERS** 3 February 1997

**Memorandum For** Turpin Ballard, USEPA Region V

**Subject:** Albion-Sheridan Township Landfill.

**1. General.** Provided below are a number of items noted during the review of the stability portion of the design analysis for the subject project.

**2. Slope Stability Analysis Methods.** There are a number of ways to evaluate the veneer stability of cover soil over geosynthetics. Two possible methods, infinite slope and force limit equilibrium analysis, are widely used in industry and can easily be performed by hand or using simple spreadsheets or computer program. Attached are two papers by Giroud et al (1995) which present both infinite slope and force limit equilibrium methods for analyzing slopes. Also attached is Report No. 18 published by the Geosynthetic Research Institute (1996) which presents a force limit equilibrium analysis method that is slightly different from that developed by Giroud et al. Stability analysis can also be performed using procedures that satisfy both moment and force equilibrium. These methods generally require a computer program for analysis.

Woodward Clyde Consultants (WWC) appears to have utilized an infinite slope type of analysis. In this type of analysis, any affects of slope buttressing at the toe are neglected. Toe buttressing improves the stability of a slope. However, neglecting toe buttressing affects is a conservative and reasonable assumption.

**3. Shear Strength Parameters.** The stability of a geosynthetically lined slope with cover soils is dependent on the slope geometry and the shear strength of the cover soils and the shear strength of the soil-geosynthetic interface. A soil's shear strength is generally quantified with two parameters; cohesion and internal friction angle. The soil-geosynthetic interface is also generally quantified with two parameters; adhesion and interface friction angle. In addition, interface shear strength parameters can be defined for "peak" or "residual" conditions. The peak shear strength is applicable to a condition where little or no displacement occurs between the geosynthetic and soil layers. Residual strengths are applicable when there is displacement between the geosynthetic and soil layers. The selection of these shear strength parameters and their use in the stability analysis has a significant impact on the calculated factor-of-safety (FS) of the slope against a sliding failure.

The geosynthetic to soil interface shear strength parameters are highly variable and are dependent on a number of variable including the type of material (e.g., HDPE or LLDPE, textured or smooth), manufacturer (i.e., different manufactures make textured geomembrane which are likely to have different interface shear strength characteristics), soil type and condition, normal stress, shearing rate, and water conditions (e.g., dry, moist, or saturated). As a result of these variables, interface shear strength parameters vary considerable from project to project. Only limited published information pertaining to the shear strength of geosynthetic interfaces is available.

WWC utilized interface friction angles for the geomembrane-soil interface of 30 and 19 degrees for the textured and smooth geomembrane cases, respectively. These values were obtained from manufacturer's data for a clean Ottawa sand to a textured HDPE geomembrane (see attached GSE data sheet). Also attached is a Technical Bulletin from Poly-Flex, Inc. which discusses interface friction. Based on Poly-Flex tests, the textured HDPE to Ottawa sand interface had a minimum friction angle of 28 degrees. The minimum interface friction angle between textured VLDPE and Ottawa sand was 25 degrees. As shown by this limited amount of data, interface friction angle are variable. Note that this data was developed for a clean medium grain sized Ottawa sand. This obviously isn't the soil that will be used to construct the landfill cap. Consequently, the appropriateness of this data needs to be addressed.

Note that WWC assumed a cover soil internal friction angle of 23 degrees for their analysis. This compares to an assumed interface friction angle of 30 degrees for the textured geomembrane case. These assumptions appear to be contradictory since the interface friction angle is greater than the internal shear strength of one of the mediums that make up the interface.

Suggest WWC provide more information and data to support the interface friction data selected for design and whether they represent peak or residual conditions. As noted above, the actual interface friction angle that can be achieved in the field is critical to the stability of the slope, particularly when seepage flows will be allowed. This is discussed in more detail in the following sections.

**4. Seepage Effects on the Stability of Slopes.** As discussed by Giroud et al (1995), "the influence of water flow on the stability of a geosynthetic-soil layered system can be very significant if the slip surface is above the geomembrane. In this case, the factor of safety of a layered system with water flow can be as low as one half of the factor of safety without water flow. The analysis shows that the influence of water is negligible or even zero, if the slip surface is below the geomembrane."

**5. WWC Soil Cover to Geomembrane Stability Analysis.** WWC performed a series of stability calculations on various interfaces in the cover system for a variety of cases. All calculations appear to have utilized assumed values for the shear strength of cover soils and of the soil to geomembrane interface. For their analysis of the 1V on 4H slopes (25%) when "the topsoil and cover soil are saturated above the relatively impervious FML and their is seepage parallel to the slope" they calculated a FS of 1.34 at the "soil-FML interface." A reanalysis of the stability of the geomembrane-soil interface on the 1V on 4H slopes based on the data used by WWC in their calculations and utilizing an infinite slope method (Giroud et al, 1995) results in a FS of 1.2 for full depth flow conditions (30-inches). Additional analysis was performed using finite slope force limit equilibrium methods where toe buttressing effects where neglected. The resultant FS from this analysis was also 1.2. A third analysis was performed using an alternative force limit equilibrium method (GRI Report 18, 1996). For this analysis, a long slope length was input to mirror infinite slope conditions. The calculated FS, which was 1.21, compared very favorable to the other methods. All stability calculations are attached. Based on these calculations, there appears to be a discrepancy between the calculated FS using the noted methods and the method utilized by WWC. This may be a result of the actual depth of seepage flow in the cover soils used in the analysis. WWC should discuss why their method of calculating the FS varies from the other noted methods.

A limited sensitivity analysis was also performed using a different interface friction angle for the soil-geomembrane interface to determine the affect on the stability of the cover system. As shown on the attached calculations, for an interface friction angle of 25 degrees, the FS drops to 1.0 for full depth flow. As demonstrated by these calculations, the FS is reduced substantially as the interface friction angle is reduced. For this reason, the assumed interface friction angle used for design needs to accurately reflect in-situ conditions in order to determine the actual FS of the slope. If the interface friction angle is overestimated, the slope may be unstable although the stability calculations would indicate otherwise (e.g., actual interface friction angle is 20 degrees however a value of 30 degrees was used in design).

As noted above, WWC should provide more data on the selection of the shear strength parameters of the geosynthetic interface. In addition, a sensitivity analysis should be performed to determine the impacts that variations in the interface friction angle and depth of seepage flow assumptions have with respect to the stability of the slope. This is especially critical since WWC is proposing to allow a significant depth of seepage to develop parallel to the slope. If the seepage depth was limited to only a few inches, there would be little concern regarding the stability of the slope.

**6. WWC Protective Soil Stability Analysis.** WWC performed an analysis of the stability of the topsoil to cover soil interface. However, they did not analyze the stability of the cover soils for a failure plane located slightly above the geomembrane in the cover soils. In essence, this is the same calculation as for the cover soil to geomembrane interface except that the soil's internal friction angle would be used instead of the interface friction angle. Based on their assumed soil internal friction angle of 23 degrees and full depth seepage (30 inches), the 1V to 4H slope would have a FS of less than one (i.e., failure). WWC should evaluate this failure mode in more detail.

**7. Required Factor of Safety.** WWC does not discuss the required FS that must be achieved for the various cases they analyzed. Suggest they provide a discussion on what is considered an acceptable FS for each case analyzed.

**8. Summary**. Slopes which are subjected to seepage flows can be much less stable than the same slope in a "dry" or non-seepage condition. The larger the flow depth above the geomembrane, the more unstable the slope becomes. For this reason, most cover systems are designed to limit the depth of seepage flows. If significant depths of seepage are allowed, the stability of the slope has to be carefully evaluated. All critical assumptions, such as interface friction angles, need to be scrutinized to assure that they are not overestimated. If shear strength data is overestimated, the slope may be much less stable than indicated by the calculations. In addition to a failure along the soil to geomembrane interface, additional evaluation of a failure in the cover soils just above the geomembrane needs to be addressed. Suggest WWC reevaluate their stability analysis based on the comments provided above.

**9. References.**


Giroud, J.P., Bachus, R.C., and Boneparte, R., 1995, "Influence of Water Flow on the Stability of Geosynthetic-Soil Layered Systems on Slopes", *Geosynthetics International*, Vol. 2, No. 6, pp 1149-1180.

Giroud, J.P., Williams, N.D., Pelte, T. and Beech, J.F., 1995, "Stability of Geosynthetic-Soil Layered Systems on Slopes", *Geosynthetics International*, Vol. 2, No. 6, pp 1115-1148.

Soong, Te-Yang, Koerner, Robert. "GRI Report 18, Cover Soil Stability Involving Geosynthetic Interfaces," Geosynthetic Research Institute, Philadelphia, PA. December 1996.

GSE FrictionFlex Application Data, 12/95

Poly-Flex, Inc. Technical Bulletin No. 101, April 1995


**10.** If you have any questions, please call me at (402) 221-3772.

Respectfully,


Richard J. Taylor, P.E.
Civil Engineer
Soils Section B, Geotechnical Branch
Engineering Division, Omaha District
U.S. Army Corps Of Engineers

| OMAHA DISTRICT | COMPUTATION SHEET | CORPS OF ENGINEERS |
|---|---|---|
| PROJECT USEPA | SHEET NO. 1 | OF 6 |
| ITEM Check of Stability Calcs | BY RJT | DATE 2/26/98 |
| | CHKD. BY | DATE |

REF: "Influence of Water Flow on the Stability of Geosynthetic-Soil Layered Systems on Slopes" Giroud, Bachus, & Bonaparte, Geosynthetics Int. Vol 2, No. 6, 1995 pp 1149–1160

• Infinite Slope Analysis, Partial Water Flow EQ.

• Above Geomembrane (i.e. Geomembrane – Soil Interface) Governing Equation

$$FS_A = \frac{\gamma_t(t-t_w)+\gamma_b t_w \tan\delta_A}{\gamma_t(t-t_w)+\gamma_{sat} t_w \tan\beta} + \frac{a_A/\sin\beta}{\gamma_t(t-t_w)+\gamma_{sat} t_w}$$ (EQ. 4 REF)

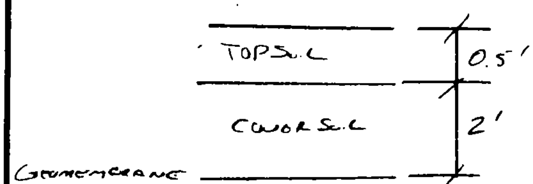
$FS_A$ = Factor of Safety Above Geomembrane
$\gamma_T$ = Total Soil Unit Weight (Moist)
$\gamma_b$ = Bouyant Soil Unit Weight ($\gamma_b = \gamma_{sat} - \gamma_w$)
$\gamma_{SAT}$ = Saturated " "
$\gamma_w$ – Water Unit Weight – 62.4 pcf
$t$ = Depth of Soil Cover
$t_w$ – Flow Depth
$\beta$ = Slope Angle
$\delta_A$ = Interface Friction Angle for Geomembrane – Soil Interface
$a_a$ = Soil Adhesion to Geomembrane

∴ Evaluate 1V on 4H case w/ Full Seepage

• 2-soil layers considered in original calc, compute average for use

$\gamma_d = 120$ pcf, $\gamma_{SAT} = 130$ pcf } Topsoil

$\gamma_d = 120$ pcf, $\gamma_{SAT} = 120$ pcf } Cover Soil




$\boxed{\gamma_{d_{AVG}} = 120 \text{ pcf} \quad \gamma_{SAT_{AVG}} = 130 \text{ pcf}}$

| OMAHA DISTRICT | COMPUTATION SHEET | CORPS OF ENGINEERS | |
|---|---|---|---|
| PROJECT USEPA | | SHEET NO. 2 | OF 6 |
| ITEM Check of Stability Calcs | | BY RJT | DATE 2/26/97 |
| | | CHKD. BY | DATE |

$$\gamma_b = \gamma_{SAT} - \gamma_w$$
$$= 130 - 62.4 = 67.6 \text{ pcf}$$

$$\boxed{\gamma_b = 67.6 \text{ pcf}}$$

$t = 2.5'$

$t_w = 2.5'$ (Full Depth Flow)

$a_a = \phi$ (No Adhesion noted in D.A. Calcs, only Cohesion. Unsure how they used. Neglecting Adhesion is conservative, Given that their design states the Interface Friction Angle = 30° for Ottawa sand, Cohesion & Adhesion should be $\simeq \phi$.

$$FS_A = \frac{\gamma_b(t-t_w) + \gamma_b t_w \tan\delta_A}{\gamma_t(t-t_w) + \gamma_{SAT}\, t_w \tan\beta} + \cancel{\frac{a_a/\sin\beta}{\gamma_t(t-t_w) + \gamma_{SAT}\, t_w}} \phi$$

$$= \frac{120(2.5-2.5) + 67.6(2.5)\tan\delta_A}{120(2.5-2.5) + 130(2.5)\tan\beta}$$

$$= 0.52 \frac{\tan\delta_A}{\tan\beta}$$

$\tan\beta = 14°$ (1V on 4H Slope)

$FS_A = 2.08 \tan\delta_A$

If $\delta_A = 30°$ (Per Design Analysis)

$FS_A = 2.08 \cdot \tan 30$

$$\boxed{FS_A = 1.20}$$

∴ This compares to 1.34 for their Cover Soil – FML Interface

∴ If $\phi = 28°$ $FS_A = 0.97$

∴ If $\phi = 23°$ (Soil) $FS_A = 0.88$

MRO Form 1550, 1 DEC 83

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3/6

# U.S. Army Corps of Engineers, Omaha District

| | | | |
|---|---|---|---|
| **PROJECT:** | EPA | **LOCATION:** | Michigan |
| **ITEM:** | Check of Stability Calcs | **DATE:** | 2/26/96 |
| File:COVSTAB3.XLS | | **BY:** | RJT |

## LIMIT EQUILIBRIUM SLOPE STABILITY ANALYSIS - SEEPAGE CASE

**Note:** Stability Calculation Based on Limit Equilibrium Analysis Method Develped in "Influence of Water Flow on the Stability of Geosynthetic-Soil Layered Systems on Slopes" by Giroud, et al, 1995

**Governing Equations:** **FS(A)=A+B+C+D+E**
**FS(B)=F+G+C+D+E**

**Where:**

A=[γt*(t-tw)+γb*tw]/[γt*(t-tw)+γs*tw]*[Tan(δa)/Tan(β)]
B=(aa/Sin(β))/[γt*(t-tw)+γs*tw]
C=[γt*(t-tw')+γb*tw']/[γt*(t-tw)+γσ*tw]*[t/h]*[Sin(ϕ)/(2*Sin(β)*Cos(β)*Cos(β+ϕ))]
D=[(c*t/h)/(γt*(t-tw)+γs*tw]*[Cos(ϕ)/(Sin(β)*Cos(β+ϕ))]
E=[(T/h)]/[γt*(t-tw)+γs*tw]
F=Tan(δb)/Tan(β)
G=(ab/Sin(β))/[γt*(t-tw)+γs*tw]

**And:**

| | |
|---|---|
| FS(A)= | Factor of Safety Above Geomembrane |
| FS(B)= | Factor of Safety Below Geomembrane |
| β= | Slope Angle |
| ϕ= | Friction angle (soil to soil) |
| δa= | Interface Friction Angle Above Geomembrane |
| δb= | Interface Friction Angle Below Geomembrane |
| γt= | Total Unit Wieght of Soil |
| γs= | Saturated Unit Wieght of Soil |
| γb= | *Bouyant Unit Wieght of Soil* |
| t= | Thickness of Soil Layer |
| tw= | Water Flow Thickness |
| tw'= | Water Flow Thickness in Wedge 1 (i.e., Toe Area) |
| aa= | Interface Adhesion Above Geomembrane |
| ab= | Interface Adhesion Below Geomembrane |
| c= | Soil Cohesion |
| h= | Slope Height |
| T= | Tension in Geosynthetics |

# U.S. Army Corps of Engineers, Omaha District

| | | | |
|---|---|---|---|
| **PROJECT:** | EPA | **LOCATION:** | Michigan |
| **ITEM:** | Check of Stability Calcs | **DATE:** | 2/26/96 |
| File:COVSTAB3.XLS | | **BY:** | RJT |

## LIMIT EQUILIBRIUM SLOPE STABILITY ANALYSIS - SEEPAGE

**CASE:** A Full Depth Flow δ = 30° NO TOE BUTTRESS

MATCHES HAND CALC

| INPUT VALUES | | | RESULTS | |
|---|---|---|---|---|
| β= | 14 degrees | 0.244 radians | A= | 1.20 |
| ϕ= | 0 degrees | 0.000 radians | B= | 0.00 |
| δa= | 30 degrees | 0.524 radians | C= | 0.00 |
| δb= | 30 degrees | 0.524 radians | D= | 0.00 |
| γt= | 120 pcf | | E= | 0.00 |
| γs= | 130 pcf | | F= | 2.32 |
| γb= | 67.6 pcf | | G= | 0.00 |
| t= | 2.5 feet | | | |
| tw= | 2.5 feet | | | |
| tw'= | 2.5 feet | | | |
| aa= | 0 psf | | **FS(A)=** | 1.20 |
| ab= | 0 psf | | **FS(B)=** | 2.32 |
| c= | 0 psf | | | |
| h= | 10 feet | | | |
| T= | 0 lbf/ft | | | |

Above Geomembrane

**CASE:** B? Full Depth Flow, δ=25°, NO TOE BUTTRESS

| INPUT VALUES | | | RESULTS | |
|---|---|---|---|---|
| β= | 14 degrees | 0.244 radians | A= | 0.97 |
| ϕ= | 0 degrees | 0.000 radians | B= | 0.00 |
| δa= | 25 degrees | 0.436 radians | C= | 0.00 |
| δb= | 25 degrees | 0.436 radians | D= | 0.00 |
| γt= | 120 pcf | | E= | 0.00 |
| γs= | 130 pcf | | F= | 1.87 |
| γb= | 67.6 pcf | | G= | 0.00 |
| t= | 2.5 feet | | | |
| tw= | 2.5 feet | | | |
| tw'= | 2.5 feet | | | |
| aa= | 0 psf | | **FS(A)=** | 0.97 |
| ab= | 0 psf | | **FS(B)=** | 1.87 |
| c= | 0 psf | | | |
| h= | 10 feet | | | |
| T= | 0 lbf/ft | | | |

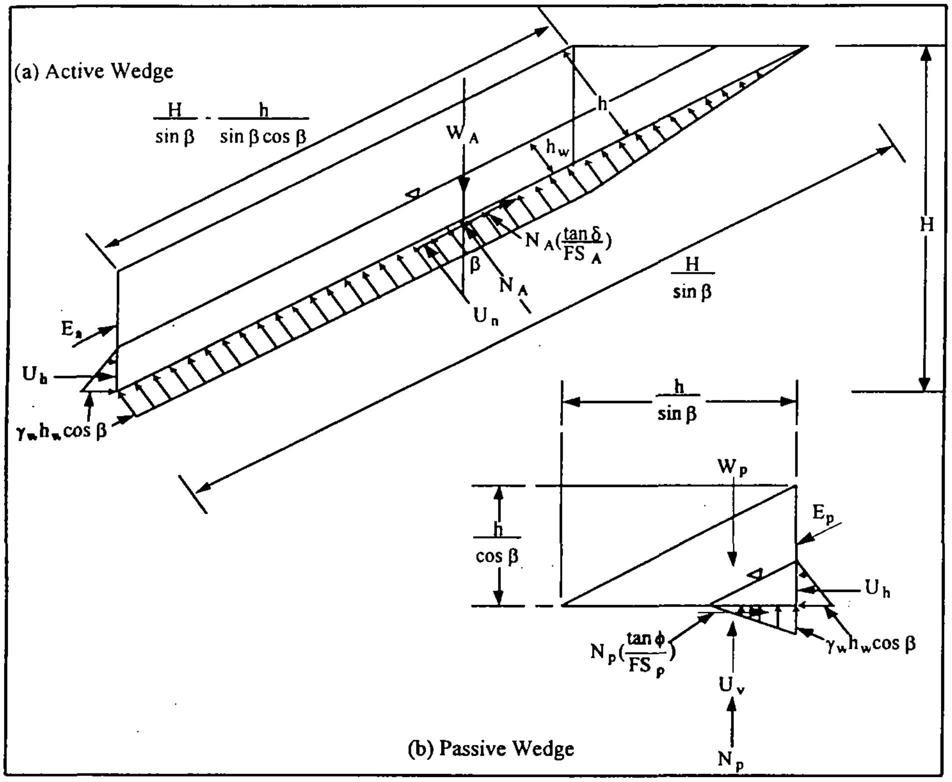
Above Geomembrane

For GSI/GRI members only

Constructed by: Te-Yang Soong

## Cover Soil Stability Analysis Worksheet for Example #5(b)

### Seepage Forces with Parallel-to-Slope Seepage Buildup




**Calculation of FS**

Active Wedge:

$WA =$ 3076 kN

$Un =$ 1435 kN

$Uh =$ 2.83 kN

$NA =$ 1549 kN

Passive Wedge:

$WP =$ 25.1 kN

$Uv =$ 11.4 kN

$$S = \frac{-b + \sqrt{b^2 - 4a}}{2a}$$

a= 722.1

b= -875

c= 0.0

**FS= 1.21**

matches Sheet 4 Case A

| | | | | |
|---|---|---|---|---|
| thickness of cover soil = h = | 0.76 | m | | |
| length of slope measured along the geomembrane = L = | 200.0 | m | | |
| soil slope angle beneath the geomembrane = β = | 14.0 | ° | = 0.24 | (rad.) |
| *vertical height of the slope measured from the toe = H =* | *48.4* | *m* | | |
| parallel submergence ratio = PSR = | 1.00 | | | |
| *depth of the water surface measured from the geomembrane = hw =* | *0.76* | *m* | | |
| dry unit weight of the cover soil = γ *dry* = | 18.9 | kN/m^3 | | |
| saturated unit weight of the cover soil = γ *sat'd* = | 20.4 | kN/m^3 | | |
| unit weight of water = γ *w* = | 9.81 | kN/m^3 | | |
| friction angle of the cover soil = φ = | 0.0 | ° | = 0.00 | (rad.) |
| interface friction angle between cover soil and geomembrane = δ = | 30.0 | ° | = 0.52 | (rad.) |

Use Length to mimic infinite slope conditions

δ = 30°

Note: numbers in boxes are input values

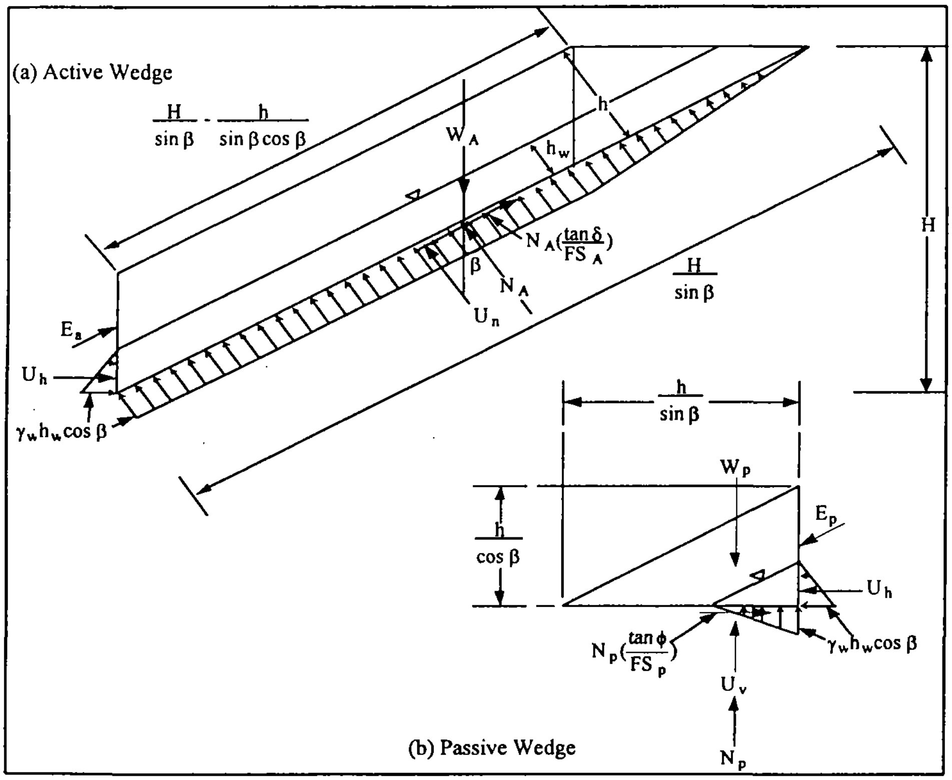
*numbers in Italics are calculated values*

For GSI/GRI members only

Constructed by: Te-Yang Soong

## Cover Soil Stability Analysis Worksheet for Example #5(b)

### Seepage Forces with Parallel-to-Slope Seepage Buildup




**Calculation of FS**

Active Wedge:

*WA = 3076 kN*
*Un = 1435 kN*
*Uh = 2.83 kN*
*NA = 1549 kN*

Passive Wedge:

*WP = 25.1 kN*
*Uv = 11.4 kN*

$$S = \frac{-b + \sqrt{b^2 - 4a}}{2a}$$

*a= 722.1*
*b= -707*
*c= 0.0*

**FS= 0.98**

Matches Sltoo 4 Case 1

| | | | |
|---|---|---|---|
| thickness of cover soil = h = | 0.76 | m | |
| length of slope measured along the geomembrane = L = | 200.0 | m | |
| soil slope angle beneath the geomembrane = β = | 14.0 | ° | *= 0.24 (rad.)* |
| *vertical height of the slope measured from the toe = H =* | *48.4* | *m* | |
| parallel submergence ratio = PSR = | 1.00 | | |
| *depth of the water surface measured from the geomembrane = hw =* | *0.76* | *m* | |

Long slope to mimic infinite slope conditions

| | | | |
|---|---|---|---|
| dry unit weight of the cover soil = γ *dry* = | 18.9 | kN/m^3 | |
| saturated unit weight of the cover soil = γ *sat'd* = | 20.4 | kN/m^3 | |
| unit weight of water = γ *w* = | 9.81 | kN/m^3 | |
| friction angle of the cover soil = ϕ = | 0.0 | ° | *= 0.00 (rad.)* |
| interface friction angle between cover soil and geomembrane = δ = | 25.0 | ° | *= 0.44 (rad.)* |

$\alpha = 25°$

Note: numbers in boxes are input values

*numbers in Italics are calculated values*